Safety requires $P_r \geq P_f$
or, $2.75P_L \leq 1385$ kN/m
Therefore, the maximum safe axial load = 1385/2.75 = 503.64 kN/m. Ans.

3-3 FLEXURE OF UNREINFORCED MASONRY

When masonry walls are subjected to transverse loading, such as wind pressure, bending takes place and, in the absence of significant axial compression, tensile stresses are introduced into the wall. If the wall is not reinforced, the tensile stresses must be resisted by the component that is weakest in tension - namely the bond between mortar and unit. Consequently, the effective area governs flexural calculations.

Because masonry is many times stronger in compression than in tension, the compressive stress is relatively low and likely to be within the linear range of the stress/strain relationship. Consequently, the flexural stresses are calculated from the well known elastic expression

$$f = \pm Mc/I = \pm M/S \qquad (3\text{-}7)$$

where

M = bending moment;
I = moment of inertia of the net section;
c = distance of the extreme fibre of the section from the centroid of the net section
= $1/2t$, where t = wall thickness;
S = elastic section modulus = I/c

If axial compression P is also present in the wall, the stresses become

$$f = P/A_e \pm M/S \qquad (3\text{-}8)$$

Bending can take place in one of two directions:

a) the wall can span vertically, as in Fig. 3-6 where it is laterally supported at the top and bottom, and tensile stresses are perpendicular to the bed (that is, horizontal) joints, as is evident from the failure mode; or,
b) the wall can span horizontally, as shown in Fig. 3-7 where it is laterally supported by vertical columns (or masonry pilasters), and tensile stresses are parallel to the bed joints. In this case the tensile effects result in a shearing action, as shown in the failure mode.

S304.1 recognizes that tensile resistance is greater in b) than in a). Table 6 of the standard gives the flexural tensile strength, both normal and parallel to the bed joints, for clay and concrete units with type S or N mortar.

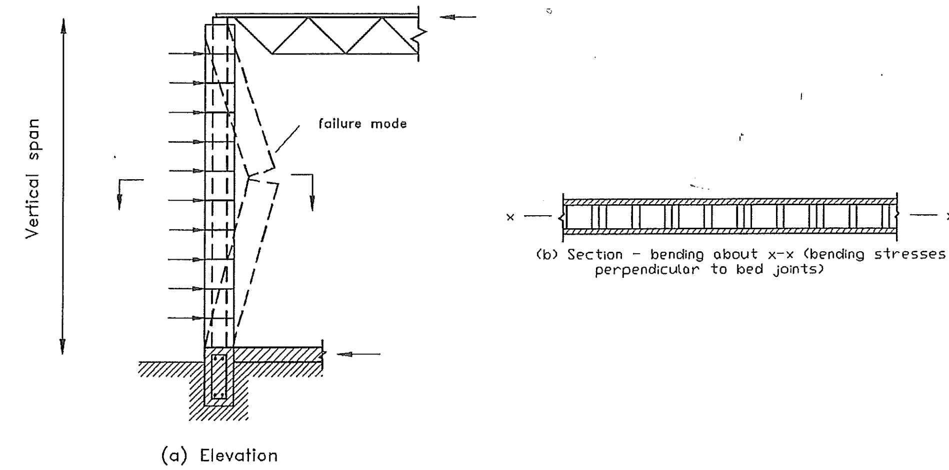

Fig. 3-6 Wall With Vertical Span

Note that in the calculation of M, simple spans are normally assumed. This is because vertical control joints (discussed in later chapters), which do not transmit moments, are likely to be present.

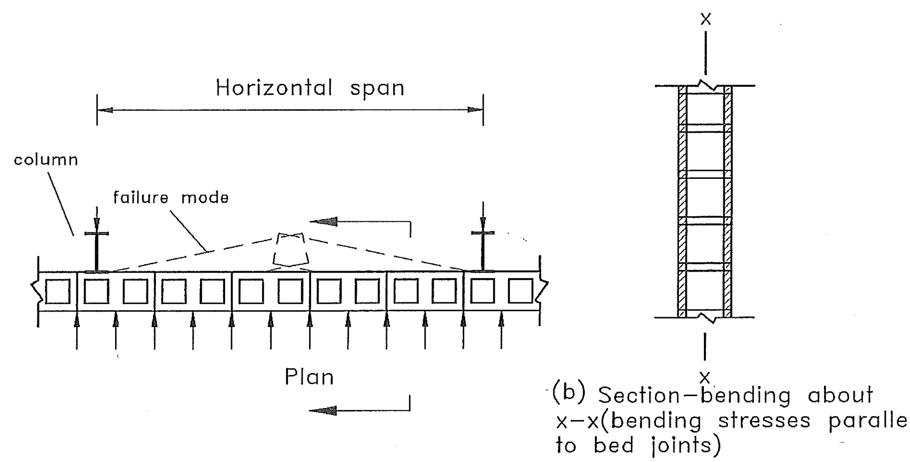

Fig. 3-7 Wall With Horizontal Span

EXAMPLE 3-5 *The unreinforced, ungrouted 200 concrete block wall shown in Fig. 3-8, spans horizontally between vertical columns spaced 4 m apart, and is subjected to a wind load of* $p = 1.0\ kN/m^2$. *If type S mortar has been used and if the mean mortar bed width is 37.7 mm, what are the stresses developed in the wall by the factored load?*

A 1.0 m strip of wall is analyzed (see Fig. 3-8) and the section effective in resisting bending is shown in Fig. 3-9, *x-x* being the axis of bending.

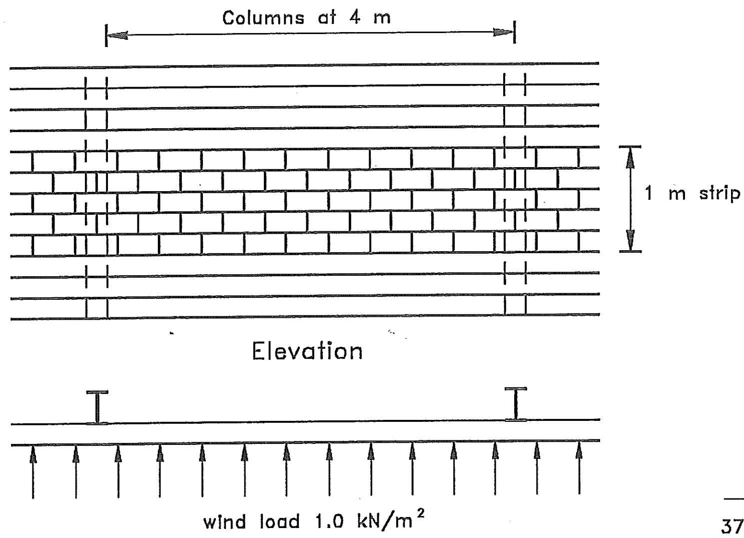

Fig. 3-8 Wall of Example 3-5

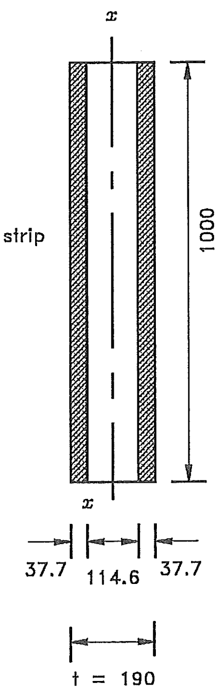

Fig. 3-9 Effective Section

For a 1.0 m strip

Recalling that the load factor $\alpha_L = 1.5$ for wind, then the factored wind is

$w_f = \alpha_L p = 1.5 \times 1.0 \text{ kN/m}^2 \times 1 \text{ m} = 1.5 \text{ kN/m}$;

span $L = 4.0$ m

Assuming simple spans, the moment is found as

$$M_x = \frac{w_f L^2}{8} = \frac{1.5(4.0)^2}{8} = 3.0 \text{ kN-m} = 3.0(10)^6 \text{ N-mm}$$

$$I_x = \frac{1000(190)^3}{12} - \frac{1000(114.6)^3}{12} = 446.2(10)^6 \text{ mm}^4$$

$$S_x = 2I_x/t = \frac{2(446.2)(10)^6}{190} = 4.70(10)^6 \text{ mm}^3$$

Therefore the bending stress is

$$f = M_x / S_x = \frac{3.0(10)^6}{4.70(10)^6} = 0.64 \text{ MPa} \qquad \underline{\text{Ans.}}$$

Note that since the wind can act in any direction, the wind stresses in the wall are = ± 0.64 MPa. From Table 6, S304.1, the limiting flexural tensile strength for this wall is f_t = 0.9 MPa and the factored tensile strength is $\phi_m f_t$ = 0.55(0.9) = 0.50 MPa. *Therefore the spacing of the columns should be reduced.*

EXAMPLE 3-6 *The free-standing 200 mm concrete block wall shown in Fig. 3-10 is unreinforced and ungrouted. If Type S mortar has been used and if the mean mortar bed width for the block is 37.7 mm, what wind pressure p, in kN/m², will produce limiting tensile stresses in the wall: (a) neglecting the self weight of the wall, and (b) including the self weight of the wall?*

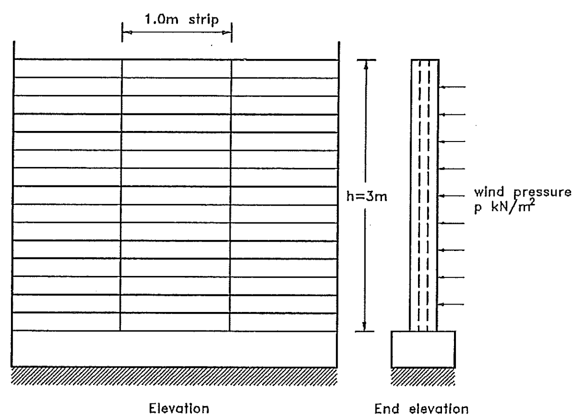

Fig.3-10 Wall analyzed in Example 3-6

<u>Note</u> 1. *The Notes on Table 6 of S304.1 limits the tensile strength f_t for free-standing cantilever walls to 0.1 MPa.*

2. *The unit weight of solid concrete masonry may be taken as 21.0 kN/m³ and the blocks may be assumed to be 50% solid.*

Since the wall is acting as a vertical cantilever, tensile stresses produced by the wind will be normal to the bed joints. The limiting tensile stress is $\phi_m f_t$ = 0.55(0.1) = 0.055 MPa.

For a 1.0 m strip

The effective section will be the same as that shown in Fig. 3-9.

[Note: The benefits of spanning in both directing and including plastic behaviour are indicated by the fact that this load of 4.06 kN/m^2 is 2.65 times the load for the wall spanning only vertically (see Example 6.3(b)). Also note that, in practice, the reinforcement in the two directions cannot occupy the same plane in the wall. Therefore, slightly different depths, and moment resistances will result in the two orthogonal directions with one depth increasing by 8 mm and the other decreasing by the same amount The lower strength for compression parallel to the bed joints (χ = 0.5) results in a larger compression zone and reduced lever arm $(d-\beta_1 c/2)$ for bending in this direction. Taking both factors into account will result in a slightly reduced capacity.]

Solution B: Using the strip method from Section 6.4.2:

$$w_f = p_v + p_h = \frac{8M_r}{h^2} + \frac{8M_r}{\ell^2}$$

$$w_f = \frac{8}{(6)^2}(6.90 + 6.90) = 3.07 \text{ kN/m}^2$$

This load is 24% lower than that predicted from yield line analysis. Generally, the strip method will provide a more conservative estimate of wall capacity compared with yield line theory.

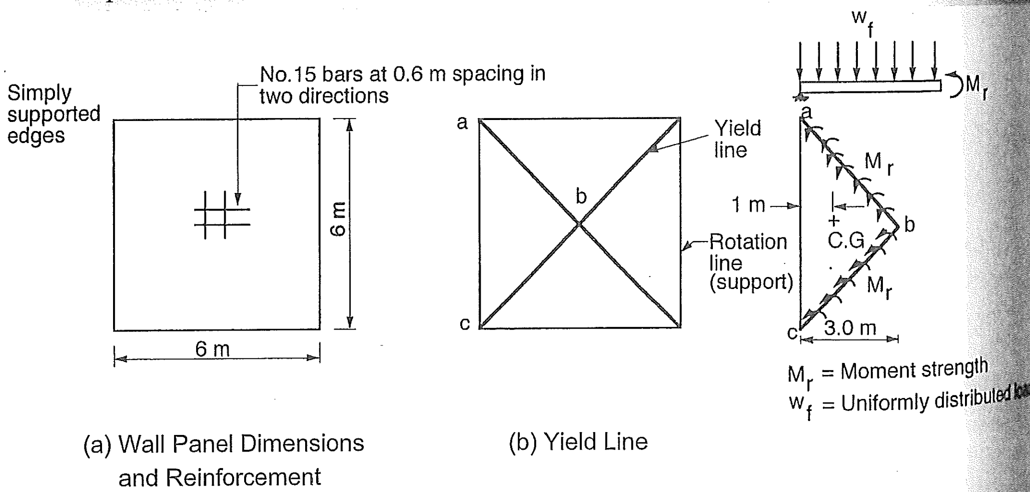

Figure 6.25 Reinforced masonry wall under two-way bending for Example 6.4.

6.8.3 Example 6.5: Horizontal Spanning Wall

Problem: Determine the amount of joint reinforcement required for a 15 cm hollow concrete masonry wall (Fig. 6.26(a)) spanning 2.4 m horizontally to carry 1.0 kN/m^2 unfactored wind load. From prism tests: f'_m = 13.8 MPa, and E_m = 10,350 MPa.

Solution: Using the load factor for wind of 1.4,

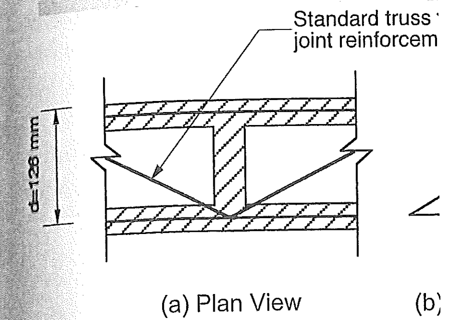

Figure 6.26 Reinforced maso

$$M_f = w_f \frac{\ell^2}{8} = \frac{1.4(1.0)(2.4)^2}{8} =$$

Try using standard truss type joint re side rod is 11.2 mm^2. Recalling that an elastic analysis may be performed 400 MPa.

Using modular ratio $n = E_s/E_m$ side rod $\rho = A_s/bd$ = 11.2/400(126) = from Eq. 5.8

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho = \sqrt{((0}$$
$$= 0.0884$$
$$j = 1 - k/3 = 0.97$$
$$M_r = \phi_s A_s f_y jd = 0.85(11.2)(40($$
$$= 0.465\times10^6 \text{ N}\cdot\text{mm/0.4 m} =$$

The strength is adequate but it is still

From Fig. 6.26(b), $\varepsilon_m = \varepsilon_y\left(\frac{kd}{d-kd}\right) =$

elastic limit at $0.5 f'_m$ of about ε_m = (1

6.8.4 Example 6.6: Design

Problem: A 4 m high nonloadbearing of 0.9 kN/m^2 and is simply suppor compressive strength of 15 MPa and

(a) as an unreinforced wall, and
(b) as a reinforced wall.

ad including plastic behaviour are indica
times the load for the wall spanning o
in practice, the reinforcement in the t
all. Therefore, slightly different depths,
ogonal directions with one depth increasi
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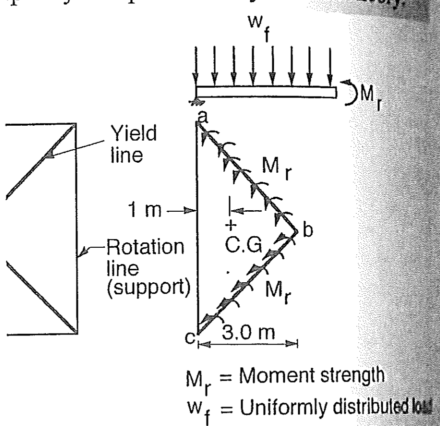

Line

two-way bending for Example 6.4.

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10,350 MPa.

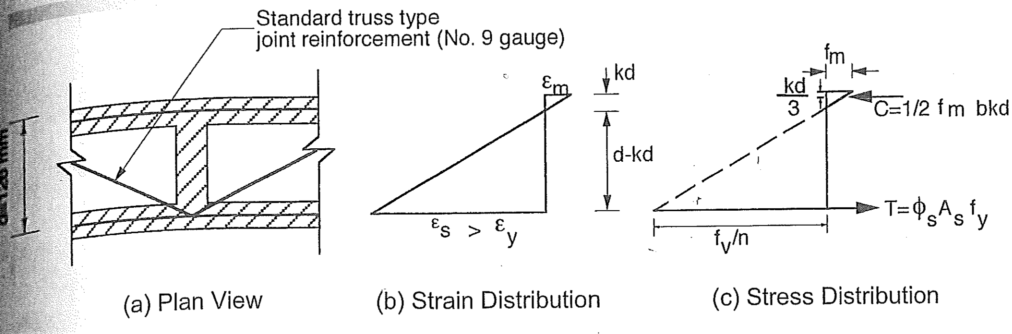

Figure 6.26 Reinforced masonry wall spanning horizontally for Example 6.5.

$$M_f = w_f \frac{\ell^2}{8} = \frac{1.4(1.0)(2.4)^2}{8} = 1.01 \text{ kN}\cdot\text{m/m}$$

Try using standard truss type joint reinforcement every second course, where the area of one side rod is 11.2 mm^2. Recalling that joint reinforcement may have limited yielding capability, an elastic analysis may be performed with the joint reinforcement just at the yield stress, f_y = 400 MPa.

Using modular ratio $n = E_s/E_m$ = 200,000/10,350 = 19.3, and steel ratio for the tension side rod $\rho = A_s/\text{bd}$ = 11.2/400(126) = 0.00022, the position of the neutral axis can be found from Eq. 5.8

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho = \sqrt{((0.00022)(19.3))^2 + 2(0.00022)19.3} - 0.00022(019.3)$$
$$= 0.0884$$
$$j = 1 - k/3 = 0.97$$
$$M_r = \phi_s A_s f_y jd = 0.85(11.2)(400)(0.97(126))$$
$$= 0.465\times10^6 \text{ N}\cdot\text{mm/0.4 m} = 1.164 \text{ kN}\cdot\text{m/m} > M_f = 1.01 \text{ kN}\cdot\text{m/m}$$

The strength is adequate but it is still necessary to check that elastic analysis is appropriate.

From Fig. 6.26(b), $\varepsilon_m = \varepsilon_y\left(\frac{kd}{d-kd}\right) = 0.002\left(\frac{0.0884}{0.9116}\right) = 0.000194$, which is well within the elastic limit at $0.5 f'_m$ of about ε_m = (13.8/2)/10,350 = 0.0008.

6.8.4 Example 6.6: Design of Nonloadbearing Wall

Problem: A 4 m high nonloadbearing concrete block masonry wall is subject to wind pressure of 0.9 kN/m^2 and is simply supported at the top and bottom. The concrete block has a compressive strength of 15 MPa and Type S mortar is used. Design

(a) as an unreinforced wall, and
(b) as a reinforced wall.